CHAPTER 3

SEISMIC DESIGN GUIDANCE FOR SHEAR WALLS (DIAGONAL STRAP SYSTEMS)

1. INTRODUCTION. The design guidance presented here is tied directly to the 1997 NEHRP (FEMA 302 and 303), because this will form the basis of the 2000 International Building Code. U.S. Army Corps of Engineers, Seismic Design for Buildings, TI 809-04 is the general military standard for seismic design of buildings, and this is also based on FEMA 302 and 303. TI 809-04 supplements the FEMA 302 and 303 with additional guidance for military buildings that is primarily based on the 1997 NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273 and 274). These and other TI 809-04 guidance that differ from the FEMA 302 and 303 are summarized in paragraph 2 of this chapter. The basis for unique seismic guidance presented here is provided in technical report found at the URL address: http://owww.cecer.army.mil/techreports/wilcfstr.post.pdf , Development of Cold-Formed Steel Seismic Design Guidance, U.S. Army Construction Engineering Research Laboratory. Design guidance is also based on the following references:

- Cold Formed Steel Design Manual, American Iron and Steel Institute, 1996 Edition.
- Manual of Steel Construction Load and Resistance Factor Design (LRFD), American Institute of Steel Construction (AISC), 2nd Edition, 1994.
- Seismic Provisions for Structural Steel Buildings, AISC, 1997.
- Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers (ASCE) 7-95, 1995.
- State-of-the-Art Report on Anchorage to Concrete, American Concrete Institute (ACI) 355.1R-91, 1991.

Unique guidance for cold-formed steel is included in this chapter, while guidance that remains unchanged from FEMA 302 and other standards is included in Appendix C as indicated in the paragraphs that follow. Appendix C also contains limited background on the development of the guidance provided in this chapter. The source of all guidance is referenced in the text. Figure 3-1 gives a flowchart for seismic design of cold-formed steel shear walls. Appendix D presents an example problem showing the application of Chapter 3 and Appendix C seismic design guidance. An update to the spreadsheet, http://owww.cecer.army.mil/techreports/wilcfsxl.post.pdf design program using the example problem is available.

2. TI 809-04, SEISMIC DESIGN FOR BUILDINGS. TI 809-04 is the military standard for the seismic design of buildings, and it provides additional guidance primarily based on FEMA 273 and 274. Additional guidance related to the design of cold-formed steel buildings is summarized as follows:

- Classification of Buildings: Definitions of Seismic Use Groups (Table 4-1) and Seismic Design Categories are expanded (Table 4-2a and 4-2b). The Seismic Use Groups are used in TI 809-04 to define Performance Objectives.
- Ground Motion: Ground Motion A is the FEMA 302 defined 2/3 site adjusted maximum considered earthquake (MCE) levels. TI 809-04 defines another ground motion level, Ground Motion B, which is defined as ¾ of the same MCE levels.
- Performance Objectives: TI 809-04 defines three performance levels: 1) Life Safety, 2) Safe Egress, and 3) Immediate Occupancy defined in Table 4-3. These levels are combined with the two design motion levels to define performance objectives for each of the four seismic use groups as described in Table 4-4. These objectives are 1A (Life Safety), 2A, 2B and 3B.
- Minimum Analytical Procedures: Chapter 5 defines three analytical procedures and the minimum procedure that must be used for each performance objective. Linear analysis with response modification factor, R as described in FEMA 302 is used for performance objective 1A. Linear analysis may also be used for Performance Objectives 2A, 2B and 3B, but with a modification factors, m for deformation controlled structural components or elements.

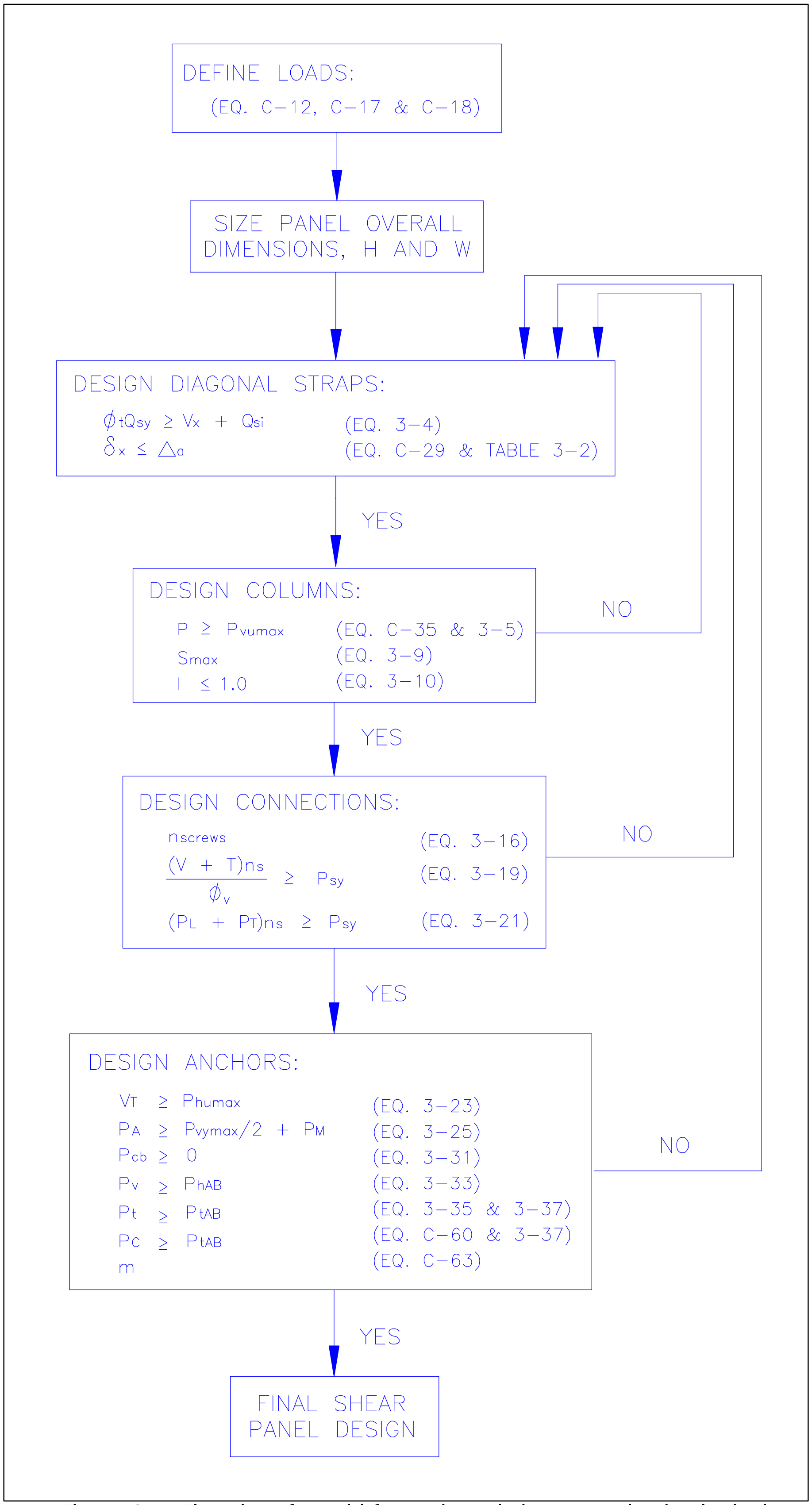

Figure 3-1. Flowchart for cold-formed steel shear panel seismic design.

- Acceptance Criteria: Acceptance criteria for each performance objective are prescribed for each analytical procedure in Chapter 6 and numerical values for each of the criteria are given in Chapters 7 through 10.

3. STRUCTURAL DESIGN CRITERIA. The basic lateral and vertical seismic-force-resisting systems considered here are diagonal strap configurations (Panels A1, A2, and D1) in Appendix B. These are considered bearing wall systems. The format of Table 5.2.2 of FEMA 302 is used in Table 3-1 to present the response modification coefficient, R and deflection amplification factor, C_d. These values are used to calculate the base shear, and design story drift. The system overstrength factor, Ω_0 used in FEMA 302 is not included here because shear panel overstrength is accounted for by $\Omega_0 Q_E$ in Equation C-16. This is the maximum lateral capacity of the shear panel based on the maximum estimated ultimate stress of the panel diagonal straps.

The response modification coefficient, R in the direction under consideration at any story shall not exceed the lowest value for the seismic-force-resisting system in the same direction considered above that story excluding penthouses. Other structural systems (dual systems) may be used in combination with these cold-formed steel panels, but then the smallest R value for all systems in the direction under consideration must be used for determining the loads applied to the entire structure in that direction. Dual systems must be used with caution, particularly if differences in stiffness result in interaction effects (FEMA 302, 5.2.2.4.2) or deformation compatibility problems (FEMA 302, 5.2.2.4.2). Another structural system may be used in the orthogonal direction with different R values, and the lowest R value for that direction only shall be used in determining loads in that orthogonal direction.

Table 3-1 Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Basic Seismic-Force-Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d	System Limitations and Building Height Limitations (ft) by Seismic Design Category				
			B	C	D	E	F
Bearing Wall Systems							
Cold-Formed Steel Shear Panels with Diagonal Strapping	4	3½	NL	NL	65	65	65

4. DEFLECTION AND DRIFT LIMITS. The design story drift, Δ shall not exceed the allowable story drift, Δ_a as obtained from Table 3-2 (FEMA 302, Table 5.2.8), for any story. The design story drift shall be computed as the difference of deflections at the center of mass at the top and bottom of the story under consideration, as determined by Equation C-29 (FEMA 302, Eq 5.3.7.1). For structures with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection, δ_x, as determined by Equation C-29, (FEMA 302, 5.2.8).

Table 3-2 Allowable Story Drift, Δ_a (mm or inches)

Structure	Seismic Use Group		
	I	II	III
Structures with diagonal strap shear walls	$0.020h_{sx}$[1]	$0.015h_{sx}$	$0.010h_{sx}$

[1] h_{sx} is the story height below level x.

5. TORSION. The distribution of lateral seismic forces shall take into account the effects of torsional moment, M_t resulting from the location of masses relative to the center of rigidity (stiffness) of the lateral force resisting frames in both orthogonal directions (FEMA 302, 5.3.5). This torsional moment shall include the effects of accidental torsional moment, M_{ta} caused by an assumed offset of the mass. This offset shall be equal to 5 percent of the dimension of the structure orthogonal to the direction of the applied seismic force. Similar to the lateral seismic forces, the torsional moments, M_t are distributed along the floors of the building according to the vertical distribution factor given in Equation C-26.

The torsional resistance comes from each of the shear wall panels, and the resistance from each panel is proportional to the square of the distance from the center of resistance to the plane of the panel. For a given panel the additional shear force due to torsion, Q_{si} can be expressed as:

$$Q_{si} = k_{si}\Delta_i = k_{si}\rho_i\theta \qquad \text{(Eq 3-1)}$$

Where:

k_{si} = the shear stiffness of shear panel i, and is defined as follows:

$$k_{si} = En_s b_s t_s \left(\frac{W}{H^2 + W^2} \right) \qquad \text{(Eq 3-2)}$$

Δ_i = the lateral in-plane shear deflection of panel i.
ρ_i = the distance from the center of resistance to panel i, perpendicular to the plane of the panel.
θ = the torsional rotation of the building at the floor level above the panel.
E = the modulus of elasticity of steel, equal to 200,000 MPa (29,000 ksi).
n_s = the number of diagonal straps.
b_s = the width of the diagonal straps.
t_s= the thickness of the diagonal straps.
W = the overall panel width.
H = the overall panel height (see Figure 3-2 for a schematic panel drawing showing W and H).

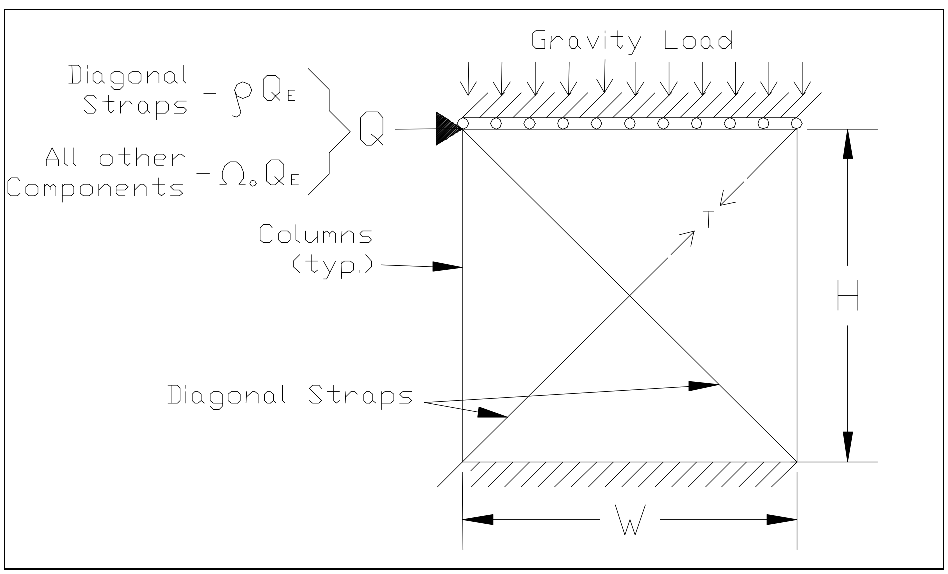

Figure 3-2. Schematic of CERL cold-formed steel shear panel model.

The torsional moment resistance, M_{tr}, for all the shear panels is given by:

$$M_{tr} = \sum_{i=1}^{n} \rho_i Q_{si} = \sum_{i=1}^{n} \rho_i^2 k_{si} \theta \qquad \text{(Eq 3-3)}$$

Equation 3-3 shows that the torsional resistance from each panel is proportional to $\rho_i^2 k_{si}$. The total torsional moment resistance, M_{tr}, is set equal to the M_t and the additional shear force due to torsion, Q_{si} is calculated using Equations 3-1 and 3-3. Note that the torsional rotation, θ in these equations does not need to be solved for and can be treated as a constant. Also the panel shear stiffness, k_{si}, is not needed if all the panels can be assumed to be equal or if their relative stiffness can be determined.

6. COLD FORMED STEEL SEISMIC REQUIREMENTS

a. Wind and Earthquake Loads. The requirements of the 1996 AISI[2], Section A5.1.3, shall be modified as follows (FEMA 302, 8.5.1): "A4.4 Wind or Earthquake Loads where load combinations specified by the applicable code include wind loads, the resulting forces are permitted to be multiplied by 0.75. Seismic load combinations shall be as determined by these provisions."

b. Boundary Members, Chords and Collectors. All boundary members, chords, and collectors shall be designed to transmit the specified induced axial forces (FEMA 302, 8.6.1). Connections for diagonal straps-to-column and columns-to-anchors and shear panel anchorage, and collectors shall have adequate strength to account for the effects of material overstrength as indicated in this guidance. The pullout resistance of screws shall not be used to resist seismic forces (FEMA 302, 8.6.2).

c. Shear Panel Anchors. Shear panels shall be anchored such that the bottom and top tracks are not required to resist uplift forces by bending of the track or track web (FEMA 302, 8.6.3). Both flanges of studs shall be braced to prevent lateral torsional buckling.

d. Pretension of Diagonal Straps. Provision shall be made for pretensioning or other methods of installation of tension-only diagonal straps, to guard against loose straps (FEMA 302, 8.6.4).

e. All Steel Design. The guidance of FEMA 302, 8.6.5 for shear walls shall not be used and configurations with plywood sheathing or oriented strand board are not permitted. The following guidance shall be used in place of the FEMA 302 guidance in section 8.6.5.

Shear panel design shall be based on the cold-formed steel shear design guidance presented here. This design requires that shear panels be adequately anchored at their top and bottoms to a floor diaphragm. Shear panels in the two orthogonal directions must be anchored to the same diaphragm at each floor level to tie the two orthogonal lateral load-resisting systems together. Shear panels above the ground floor must have shear panels in the same bay and direction at every floor level below them.

Using the following guidance, the diagonal straps are sized to resist the total horizontal loads at each floor level as defined in Equations C-12 and C-13, based on trial shear panel locations and aspect ratios. Then the greater loads defined in Equations C-17 and C-18 are used to size the shear panel columns. Finally the panel connections and anchors are designed based on the guidance that follows.

[2] Specification for the Design of Cold-Formed Steel Structural Members, American Iron and Steel Institute (AISI), 1996.

7. DIAGONAL STRAP DESIGN. The diagonal straps are designed to resist the seismic story shears, V_x given in Equation C-27 that has been increased by the additional shear force due to torsion (Q_{si} in Equation 3.1). The shear panels shall be configured and diagonal straps sized so that the lateral shear panel design strength, $\phi_t Q_{sy}$ satisfies the following equation (see Equation C-34 in Appendix C).

$$\phi_t Q_{sy} = \phi_t \sum_{i=1}^{n} \left[n_s b_s t_s F_{sy} \left(\frac{W}{\sqrt{H^2 + W^2}} \right) \right] \geq V_x + Q_{si} \qquad \text{(Eq 3-4)}$$

Where:

ϕ_t = the resistance factor for tensile members (0.95),
n = the number of shear panels in the building frame for which the shear forces V_x and Q_{si} are applied.
n_s = the number of diagonal straps (panel faces with straps) in an individual panel.
F_{sy} = the design yield strength of the strap.

The number of shear panels, panel width, height, and strap size and strength shall be designed according to Equation 3-4 to meet minimum lateral yield capacity. All diagonal strap material must be ASTM A653 steel. Diagonal straps may not use re-rolled steel, because the re-rolling strain hardens the material, increasing material strength variability and reducing elongation (see USACERL Technical Report, Chapter 4 for a discussion of this concern).

8. COLUMN DESIGN – Structural Tubing or Built-up from Studs. The columns of the Panel A configuration are built-up with cold-formed steel studs. These studs must be oriented to form a closed cross-section as shown on the Test Panel A1 and A2 drawings in Appendix B. Individual studs must be welded to each other with a weld thickness equal to the thickness of the studs. The welds are intermittent, with a length and spacing that will ensure composite behavior of the column.

Structural tubing column design (Panel D configuration - Drawing D1 in Appendix B) follows the same procedure, but consists of a single member which is a closed section by itself. The equations in this guidance are used such that the number of studs making up this column is one.

a. Column Applied Loads. Loads applied to the columns are defined based on Equation C-17 (TI 809-04, Equations 4-2 and 4-6), where the effects of gravity load and seismic forces are additive and diagonal strap overstrength is accounted for. Only that portion of gravity loads applied to the tributary area of the shear panel columns are included in the design of these columns. However, the full horizontal seismic force, $\Omega_0 Q_E$ applied to the shear panel and resisted by the diagonal straps, will add a vertical component to the columns, increasing axial load. This horizontal load is based on the actual designed area of the diagonal straps as defined in Equation C-16. The total column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax} is:

$$P_{vu\max} = \frac{GL_{\max}}{2} + F_{su\max} n_s b_s t_s \left(\frac{H}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq 3-5)}$$

Where:

GL_{max} = the maximum gravity load per shear panel, i.e., from $(1.2 + 0.2S_{DS})D + 0.5L + 0.2\,S$, in Equation C-17.
F_{sumax} = the maximum estimated ultimate stress in the diagonal straps, which equals to 1.5 F_{su} for ASTM A653 Grade 33 steel (F_{su} = 310 MPa and 45 ksi) and 1.25 F_{su} of Grade 50 steel (F_{su} = 448 MPa and 65 ksi).

b. Column Axial Capacity. Column capacity is determined based on AISI provisions. The design strength, P shall be determined based on the AISI guidance (Section C4 Concentrically Loaded

Compression Members). This guidance, applied to columns built-up with cold-formed steel studs or individual structural tubing members, is summarized in Appendix C (Paragraph C13). Columns shall be designed such that their design strength, P (Equation C-35) exceeds the total axial applied load, P_{vumax}.

c. Column Bending Load and Composite Behavior. The column anchor design provisions developed later in this guidance will create a moment connection. The primary purpose of the anchor design is to resist shear and uplift forces. However, this anchor design will also allow the columns to act as a moment frame, providing limited structural redundancy and widening of the hysteretic load deflection envelopes of the shear panel. This will allow the panels to absorb more energy under cyclic seismic loading conditions. The columns built-up from studs must be designed to act as a composite cross section in order to provide this moment capacity. This will require welding between the studs that will provide the shear transfer needed to develop the maximum moment in the columns. When one diagonal strap is in tension, the full gravity load on the shear panel may be carried in a single column, with the other column having no axial load. The maximum moment in a column will occur when it has no axial load. Therefore the welds shall be designed for the full moment capacity of the columns. This design requirement will allow the shear panel columns to continue providing bending resistance beyond the lateral yield deflection of the columns. These welds shall resist the maximum shear between the studs, which will be between the studs closest to the column neutral axis. This shear, q is defined as follows:

$$q = \frac{V_c Q}{I_c} \qquad \text{(Eq 3-6)}$$

Where:

V_c = the maximum column shear due to column moment only.

Q = the moment of the column cross-sectional area on one side of the critical weld about the critical weld plane.

I_c = the moment of inertia of the column due to bending in the plane of the shear panel.

The maximum column shear, V_c due to the maximum column moment M_c only is determined as follows:

$$V_c = \frac{2M_c}{H} = \frac{2F_{cy}I_c}{Hc} \qquad \text{(Eq 3-7)}$$

Where:

H = the panel height

F_{cy} = the yield strength of the column. This strength is not increased for column material overstrength because weld failure is controlled by the column material strength, so that any material overstrength would result in a proportionately greater weld strength.

c = the distance to the column neutral axis to the extreme fiber in the plane of the shear panel.

The moment of the column cross-sectional area on one side of the critical weld about the critical weld plane, Q is defined as follows:

$$Q = \int_A y\,dA = A\bar{y} \qquad \text{(Eq 3-8)}$$

Where:

A = the area of column cross-section on one side of the critical weld plane closest to the column neutral axis.

$\bar{y}$ = the distance from the neutral axis of the column cross-sectional area on one side of the critical weld plane to the critical weld failure plane.

Built-up columns are fabricated by welding individual studs together to form a closed cross-section, using flare V-groove welds. The same weld size and spacing shall be used between all studs in the built-up column. These welds are design according to AISI (Section E2.5 Flare Groove Welds), assuming double shear. The maximum spacing between centers of intermittent welds, s_{max} is determined as follows:

$$s_{max} = 1.5\phi_G t_c F_{cu} \frac{L}{q} \qquad \text{(Eq 3-9)}$$

Where:

ϕ_G = the resistance factor for flare grove welds, equal to 0.55.
t_c = the stud thickness of the built-up columns.
F_{cu} = the ultimate strength of the column steel.
L = the length of intermittent grove welds.
q = the maximum shear determined in Equation 3-6.

Intermittent welds shall be made at both the top and bottom ends of the columns, regardless of the maximum center-to-center spacing, s_{max}.

d. Column Combined Axial and Moment Capacity. The combination of axial load and bending shall be evaluated using a modification to AISI guidance (C5.2.2 Combined Compressive Axial Load and Bending – LRFD Method). The combination of axial and moment on the column shall be evaluated based on the following interaction equation (modification of AISI Equation C5.2.2-2):

$$I = \frac{P_{vu\,max}}{F_{cy} A_c} + \frac{M_a}{M_{nx}} \leq 1.0 \qquad \text{(Eq 3-10)}$$

Where:

P_{vumax} = the applied axial load, defined in Equation 3-5.
A_c = the nominal column cross-sectional area.
M_a = the applied moment at maximum estimated strap yield strength, defined in Equation 3-11. This equation conservatively assumes the column is fully fixed at its top and bottom by the panel anchors. This moment is also conservatively based on the maximum panel lateral deflection at which the diagonal strap will yield. This moment includes the column bending and P-delta effect of axial load. Still, this moment will be less than the column moment with no axial load (paragraph 3-8c). The applied moment, M_a is defined as follows:

$$M_a = \frac{6EI_c \delta_{sy\,max}}{H^2} + P_{vu\,max}\delta_{sy\,max} \qquad \text{(Eq 3-11)}$$

Where:

δ_{symax} = the maximum estimated lateral panel deflection at the maximum estimated yield strength of the diagonal straps, F_{symax} and is defined as follows:

$$\delta_{sy\,max} = \frac{F_{sy\,max}}{E}\left(\frac{H^2 + W^2}{W}\right) \qquad \text{(Eq 3-12)}$$

Where:

F_{symax} = maximum estimated yield stress of the diagonal straps, equal to $2F_{sy}$ for Grade 33 and $1.5F_{sy}$ for Grade 50 steel.

M_{nx} = the column gross cross-section nominal moment capacity, and this is defined as follows (modification of AISI Equation C3.1.1-1):

$$M_{nx} = F_{cy}\left(\frac{I_c}{h_c - c}\right) \qquad \text{(Eq 3-13)}$$

Where:

h_c = the width of the column in the plane of the shear panel.
c = the distance from the column neutral axis to the extreme fiber.

e. Column Shear Capacity. The trial column design must be checked for shear capacity. The diagonal straps fasten to the columns near their connection to the tracks and column anchor. Therefore the column must either have adequate shear capacity for the maximum horizontal seismic force, $\Omega_0 Q_E$ applied to the shear panel, or the column shear capacity must be augmented with other components. The column shear design strength, V_C shall be determined based on AISI guidance (Section C.3.2, Strength for Shear Only). This guidance, applied to columns built-up with cold-formed steel studs or individual structural tubing members, is summarized in Appendix C (Paragraph C14). For columns built-up with studs, the maximum stud flange width over thickness, $(h/t)_{max}$ is defined as follows:

$$\left(\frac{h}{t_c}\right)_{max} = 0.96\sqrt{\frac{Ek_v}{F_{cy}}} \qquad \text{(Eq 3-14)}$$

Where:

h = the depth of the flat portion of the column web, which equals the stud flange width.
t_c= the column web thickness, which equals the stud thickness.
k_v = the shear buckling coefficient, which equals 5.34.

For studs with a flange width of 50-mm (2 inches) this requires a minimum stud thickness of 0.77-mm (30 mil or 20 gauge) for 33 ksi steel and 0.95 mm (37 mil or 18 gauge) for 50 ksi steel.

Columns have insufficient shear capacity by themselves, and require additional shear capacity from their anchorage detail (see paragraph 3.10a for anchorage shear design guidance).

9. CONNECTION DESIGN.

a. Connection Design Assumptions and Applied Loads. This paragraph provides connection design assumptions that define loading and load path issues for cold-formed steel shear panels. These assumptions apply to the diagonal strap-to-column connections. These loads are based only on the maximum lateral force, $\Omega_0 Q_E$. This force results from the right-hand term in Equation C-18, $\Omega_0 Q_E$, which accounts for diagonal strap material overstrength. The maximum estimated ultimate force in the diagonal straps (in the axis of the straps), P_{su}, is:

$$P_{su} = F_{su\,max} n_s b_s t_s \qquad \text{(Eq 3-15)}$$

The diagonal strap-to-column connection shall be designed to resist the force defined by Equation 3-15.

Panel design will require the use of angle section anchors as described under panel anchors (pargraph 10), because of the shear transfer requirements. This anchor will also transfer loads between the column and base or top beam, or floor slabs, thereby eliminating the need for load transfer with a column-to-track connection. In low seismic zones it may be possible to transfer the shear forces with a column-to-track connection only, without anchors. However, it is considered more reasonable to use fewer shear panels rather than many with low lateral load capacity. Therefore all shear panel design guidance presented here shall require the use of anchors. Anchor design is presented later in this document (Paragraph 10).

b. Screwed Fastener Connection Design. Self-tapping screwed connection capacity definition shall follow AISI guidance (Section E4 Screw Connections). Screws shall be installed and tightened in accordance with the manufacturer's recommendations. Screw connections loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pullout of the screw, and bearing of the joined materials. The commentary of the AISI Specification (E4.3) gives further explanation and illustration of these modes of failure. The AISI provisions focus on the tilting and bearing modes of failure. Two cases are given depending on the ratio of the connected member thicknesses. Normally the head of the screw will be in contact with the thinner material, t_1. However, when both materials are the same thickness or the thicker member is in contact with the screw head, tilting becomes a more critical mode of failure. The AISI Section E4 guidance on design shear strength per screw, P_s applied to diagonal strap-to-column screw connections is summarized in Appendix C (Paragraph C15). The modes of failure expressed in Equations C-48 through C-52 are defined alongside the equations.

Minimum spacing guidance (AISI E.4.1) requires that the distance between centers of fasteners shall not be less than 3d, where d is the nominal screw diameter.

Minimum edge and end distance guidance (AISI E.4.2) requires that the distance from the center of a fastener to the edge of any connected part shall not be less than 3d. If the connection is subjected to shear force in one direction only, the minimum edge distance shall be 1.5d in the direction perpendicular to the force.

Finally, the minimum number of screws required at each diagonal strap-to-column connection, n_{screws} is calculated as follows:

$$n_{screws} \geq \frac{P_{su}}{n_s P_s} \quad \text{(Eq 3-16)}$$

The nominal shear strength of the screws shall be determined based on manufacturer's data (AISI E4.3.2), which must be based on tests according to AISI Section F1(a). This nominal shear strength of approved screws must not be less than $1.25P_{ns}$ where P_{ns} is defined by Equations C-48 through C-52.

c. Design Rupture Strength. The design shear strength along a potential shear rupture plane between fasteners of connected members, V shall be determined as below (AISI E5 Shear Rupture, Specification and Commentary, AISC J4 Design Rupture Strength). The AISI commentary states that their shear rupture provisions are based on AISC provisions. The AISI provisions conservatively neglect the greater strength that AISC allows for the tensile rupture plane. The guidance below adds the greater strength allowed by AISC for this tensile rupture plane.

$$V = 0.6\phi_v F_u A_{nv} \quad \text{(Eq 3-17)}$$

Where:

ϕ_v = the shear rupture resistance factor, equal to 0.75.
F_u = the ultimate strength of the member being evaluated.
A_{nv} = the net area subjected to shear along the rupture plane being considered.

The design tensile strength along a potential tensile rupture plane between fasteners of connected members, T shall be determined as follows:

$$T = \phi_t F_u A_{nt} \quad \text{(Eq 3-18)}$$

Where:

ϕ_t = the tensile rupture resistance factor, equal to 0.75
A_{nt} = the net area subjected to tension along the rupture plane being considered.

The shear and tensile rupture strength are based on the diagonal strap ultimate strength of the member in the joint being evaluated. The maximum applied load on this joint is based on the yield strength of the same member, P_{sy}. This will be much less than the maximum estimated strap axial force, P_{su}. The maximum force in the members is not critical, but rather the minimum ratio of F_u/F_y because the rupture strength capacity is dependent on F_u and the maximum applied force is dependent on F_y. This guidance requires that ASTM A653 material be used for the straps and the minimum F_u/F_y ratio for Grade 33 and Grade 50 material is 1.36 and 1.30[3], respectively. These minimum ratios equate to yield, F_y and ultimate strengths, F_u of Grade 33 and Grade 50 material, such that F_{y33} = 228 MPa (33 ksi), F_{u33} = 310 MPa (45 ksi), F_{y50} = 345 MPa (50 ksi), and F_{u50} = 448 MPa (65 ksi). Therefore the strap yield strength, P_{sy} may be defined simply based on the yield strength of these materials. This requirement is expressed as follows:

$$(V + T)n_s \geq P_{sy} \quad \text{(Eq 3-19)}$$

Where:

$$P_{sy} = F_y n_s b_s t_s \quad \text{(Eq 3-20)}$$

When the strap-to-column rupture strength is evaluated based on Equation 3-19, the resistance factors in Equations 3-17 and 3-18 may be increased to 1.0, because of the ASTM minimum material requirement on F_u/F_y stated above.

d. Welded Connection Design. Welded design shall follow AISI guidance (Section E2 Welded Connections). This guidance covers connections of members in which the thinnest member is 0.18 inches or less. Arc welds shall be made in accordance with AWS D1.3[4] and its commentary. Resistance welds shall be made in accordance with the procedures in AWS C1.1 or AWS C1.3.

Welded diagonal strap-to-column connections will require fillet welds (AISI E2.4). The welds at the sides of the straps will be loaded in the longitudinal direction, and welds at the ends of the straps will be loaded in the transverse direction. The weld thickness should be equal to the thickness of the strap material. Ultimate failure of fillet welded joints has usually been found to occur by the tearing of the plate adjacent to the weld. The higher strength of the weld material prevents weld shear failure, therefore, this guidance is based on sheet tearing.[5] Fillet weld design for diagonal strap-to-column connections is summarized in Appendix C (Paragraph C16).

The fillet weld longitudinal and transverse shear strengths are based on the ultimate strength of the thinner member (normally diagonal strap) of the joint. The maximum applied load on this joint is based on the yield strength of the same member, P_{sy}. The maximum force in the members is not critical, but rather the minimum ratio of F_u/F_y because the rupture strength capacity is dependent on F_u and the maximum applied force is dependent on F_y. This guidance requires that ASTM A653 material be used for the straps and the minimum F_u/F_y ratio for Grade 33 and Grade 50 material is 1.36 and 1.30[6], respectively. Therefore, the strap yield strength, P_{sy} shall be defined simply based on the yield strength of these materials. This requirement is expressed as follows:

[3] See AISI Dimensions and Properties for use with the 1996 AISI Cold-Formed Steel Specifications, ASTM Specifications for Referenced Steels.
[4] American Welding Society (1989), Structural Welding Code - Sheet Steel, ANSI/AWS D1.3-89, Miami, FL 1989.
[5] AISI Commentary, E2.4.
[6] See AISI Dimensions and Properties for use with the 1996 AISI Cold-Formed Steel Specifications, ASTM Specifications for Referenced Steels.

$$(P_L + P_T)n_s \geq P_{sy} \qquad \text{(Eq 3-21)}$$

10. PANEL ANCHORS. Panel anchors must be installed on both sides of the shear panel columns because the columns by themselves have inadequate shear capacity. Furthermore, if the column were simply fastened to the track, the track would be loaded in bending, due to uplift. The track is very weak in bending and this would violate the guidance stated in Paragraph 6c. Therefore, anchors consisting of angle iron sections shall be welded to both sides of the column at both the top and bottom of the columns to provide the required panel anchorage. Loose steel plates are laid over the horizontal portion of the angle sections. The angles and plates shall be drilled with through holes and anchored to the supporting diaphragm above and below the shear panel using embedded anchor bolts. See Appendix D (Figures D4 through D9) for examples of this anchor configuration.

a. Anchor Shear Capacity. Columns have insufficient shear capacity by themselves, and require additional shear capacity from their anchorage detail. This will require the installation of angle iron anchors on both sides of the columns, such that one leg of the angle extends beyond the critical shear plane. For screwed fastener connections, the critical shear plane is along the horizontal row of screws closest to the track in the diagonal strap-to-column connection. For the welded connections, the critical shear plane is along the strap-to-column weld near the track. The angle iron anchor shear design strength, V_A for a single angle is defined as follows:

$$V_A = 0.6\phi_v F_{Ay} b_c t_A \qquad \text{(Eq 3-22)}$$

Where:

$\phi_v = 1.0.$
F_{Ay} = the anchor angle iron yield strength.
b_c = the width of the angle, which equals the out-of-plane width of the column.
t_A = the thickness of the angle.

The total design shear strength, V_T must exceed the maximum shear panel horizontal seismic force P_{humax} ($\Omega_0 Q_E$). All anchors are made up with two angles, on either side of the column, so that V_T may be expressed as:

$$V_T = V_C + 2V_A \geq P_{hu\,max} \qquad \text{(Eq 3-23)}$$

Where:

V_C = the column shear capacity determined in according to Equation C-46 in Appendix C.

$$P_{hu\,max} = \Omega_0 Q_E = F_{su\,max} n_s b_s t_s \left(\frac{W}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq 3-24)}$$

b. Anchor Angle and Plate Design. The most critical load condition for anchors is when the effects of gravity load and seismic forces counteract each other. This load condition is expressed by Equation C-18.

The selected angle and plate anchors shall resist the applied shear and uplift forces. These anchors will also provide limited moment resistance. The angles and plates will yield in flexure between the anchor bolts and bend in the angle, but will not fail in a brittle manner. This limited moment resistance will slightly widen the hysteretic envelope in the load deflection performance of the panel. The angles and plates can yield significantly through many cycles with no loss of shear and uplift resistance (some loss of moment resistance). The maximum weld thickness to the column shall be used, which is based on the thickness of the column material, as indicated in Table 3-3. The panel anchors shall be constructed using angle sections with a thickness equal to the maximum permitted based on the column-to-anchor weld thickness (see Table 3-4).

The limitation on angle thickness will cause the angle to yield in bending at the angle corner, so that it provides little resistance to uplift by itself. Uplift resistance shall be increased by adding a plate over the horizontal leg of the angle.

Table 3-3. Maximum Column-to-Anchor Weld Thickness.[7]

Column Material Thickness, t_c	**Maximum Weld Thickness, t_w**
t_c < 6 mm (¼ inch)	$T_w = t_c$
$t_c \geq$ 6 mm (¼ inch)	$t_w = t_c$ – 1.5 mm (t_c – 1/16 inch)

Table 3-4. Maximum Angle Thickness Based on Column-to-Anchor Weld Thickness.[8]

Weld Thickness, t_w	**Maximum Angle Thickness, t_A**
3 mm (1/8 inch)	6 mm (1/4 inch)
5 mm (3/16 inch)	13 mm (1/2 inch)
6 mm (1/4 inch)	19 mm (3/4 inch)
8 mm (5/16 inch)	29 mm (1-1/8 inch)[9]

The column-to-angle weld design strength, P_A shall exceed the total uplift force applied to one angle at one side of the column due to uplift and bending. This is expressed as follows:

$$\frac{P_{vy\,max}}{2} + P_M \leq P_A = P_T + P_G \qquad \text{(Eq 3-25)}$$

Where:

P_A = the total vertical design capacity of the column-to-angle weld

P_T = the design strength of the transverse loaded fillet weld at the horizontal column-to-angle weld (Equation C-56)

P_G = the design strength of the longitudinal loaded flare bevel grove weld at the vertical column-to-angle welds at the corner of the columns. The design strength for this column-to-angle weld shall be determined based on AISI guidance (Section E2.5 Flare Grove Welds). The application of this guidance to the design of column-to-angle welds is summarized in Appendix C (Paragraph C17).

P_{vymax} = the net anchor vertical load at the maximum yield stress in the diagonal straps, expressed by:

$$P_{vy\,max} = F_{sy\,max} n_s b_s t_s \left(\frac{H}{\sqrt{H^2 + W^2}} \right) - \frac{GL_{min}}{2} \qquad \text{(Eq 3-26)}$$

Where:

GL_{min} = the minimum gravity load per shear panel, i.e., (0.9 - 0.2S_{DS})D in Equation C-18.

P_M = the uplift force capacity per anchor angle, beyond P_{vymax}/2 available to resist moment is determined by Equation 3-27. This assumes the anchor bolts are sufficiently tightened to provide a moment restraint.

[7] AISC Load and Resistance Factor Design (LRFD) Specification, 2nd Edition, 1994, Section J2b.
[8] AISC LRFD, Table J2.4
[9] Maximum thickness of standard angles.

$$P_M = \frac{M_A - \frac{P_{vy\,max}}{2}\frac{d_b}{2}}{\frac{d_b}{2}} \qquad \text{(Eq 3-27)}$$

Where:

M_A = the plastic moment capacity of the angle and plate resting over the horizontal leg of the angle

d_b = the distance from the plate edge where the angle corner begins to the critical bending plane in the plate. The critical bending plane is at the edge of the anchor bolt nut(s) nearest to the columns.

The plastic moment capacity of the angle and plate, M_A is calculated as follows:

$$M_A = \phi_b F_{Ay} \frac{b_c}{4}\left(t_A^2 + t_p^2\right) \qquad \text{(Eq 3-28)}$$

Where:

ϕ_b = the bending resistance factor, equal to 0.90.
F_{Ay} = the yield strength of the angle and plate.
b_c = the length of the angle, which equals the anchor width and out-of-plane width of the column.
t_A = the thickness of the angle.
t_p = the thickness of the plate.

The distance from the plate edge to the critical bending plane, d_b is determined as follows:

$$d_b = d_c - k - \frac{W}{2} \qquad \text{(Eq 3-29)}$$

Where:

d_c = the distance from the center of anchor bolts to the column face.
k = the distance from the corner of the angle to the flat portion of the angle legs (from AISC LRFD, Dimensions and Properties of Structural Shapes).
W = the width across flats of the anchor bolt nut(s). This dimension is given in AISC LRFD, Volume II Connections, Table 8-2, Dimensions of High-Strength Fasteners.

The column moment connection capacity, M_c is defined as follows:

$$M_c = P_M(h_c + t_A + k) \qquad \text{(Eq 3-30)}$$

Where:

h_c = the depth or in-plane width of the column.

A portion of this moment is used to resist the moment created by the eccentric loading of diagonal strap-to-column connection with respect to the center of the column anchor, $P_{symax}L_s$. The angle uplift capacity that remains to resist column bending, P_{cb} shall be greater than zero and is determined as follows:

$$P_{cb} = \frac{M_c - P_{sy\,max}L_s}{h_c + t_A + k} \qquad \text{(Eq 3-31)}$$

Where:

P_{symax} = the maximum yield strength of the diagonal strap(s) in the shear panel, in the axis of the strap. This is determined as follows:

$$P_{sy\,max} = F_{sy\,max} n_s b_s t_s \qquad \text{(Eq 3-32)}$$

L_s = the diagonal strap eccentricity equal to the distance from the center of the diagonal strap-to-column connection to the center of the column-to-anchor connection, perpendicular to the axis of the diagonal strap.

c. Anchor Bolt Design. The anchor bolts that fasten the column anchors to the reinforced concrete beam or slab are next designed. The same detail used in the anchors at the base of the column shall be used in the anchor at the top of the column. The anchor bolts shall be sized based on the bolt shear strength, P_v tensile strength, P_t and cone failure design strength, P_c. The anchor bolt shear design strength, P_v shall exceed the applied shear load per bolt, P_{hAB}. This is expressed as follows:

$$P_v \geq P_{hAB} = \frac{P_{hu\,max}}{n_{AB}} \qquad \text{(Eq 3-33)}$$

Where:

$$P_v = \phi_{tv} F_v \frac{\pi}{4} d_{AB}^2 \qquad \text{(Eq 3-34)}$$

n_{AB} = the number of anchor bolts in the anchor on both sides of the column
ϕ_{tv} = the tensile and shear resistance factor (0.75[10]).
F_v = the nominal shear strength of the anchor bolts.[11]
d_{AB} = the diameter of the anchor bolt.
P_{humax} = the maximum shear panel horizontal force defined by Equation 3-24.

The anchor bolt-tensile design strength, P_t shall exceed the applied tensile force per bolt, P_{tAB}. The anchor bolt tensile strength, P_t[12] is determined as follows:

$$P_t = \phi_{tv} F_t \frac{\pi}{4} d_{AB}^2 \qquad \text{(Eq 3-35)}$$

Where:

F_t = the nominal tensile strength of the anchor bolts determined by the minimum value given in AISC LRFD, Tables J3.2 (Design Strength of Fasteners) and J3.5 (Tension Stress Limit (F_t) for Fasteners in Bearing-type Connections. The value of f_v used in Table J3.5 is determined as follows:

$$f_v = \frac{P_{hAB}}{\frac{\pi}{4} d_{AB}^2} \qquad \text{(Eq 3-36)}$$

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[10] AISC LRFD, Table J3.2, Design Strength of Fasteners.
[11] AISC LRFD, Table J3.2, Design Strength of Fasteners.
[12] American Concrete Institute (ACI), Manual of Concrete Practice, Part 3, State-of-the-Art Report on Anchorage to Concrete – ACI 355.1R-91, 1991, equation 3.1.

The applied tensile force per anchor bolt, P_{tAB} is calculated as follows:

$$P_{tAB} = \frac{(P_{cb} + \frac{P_{sy\,max} L_s}{h_c + t_A + k} + \frac{P_{vy\,max}}{2})(W_A - \frac{t_A}{2})}{(W_A - d_c)(\frac{n_{AB}}{2})} \qquad \text{(Eq 3-37)}$$

The anchor bolt cone failure design strength, P_c shall exceed the applied tensile force per bolt, P_{tAB}. The anchor bolt cone failure design strength, P_c is determined based on the guidance in ACI 355.1R-91.[13] The applications of this guidance to anchor bolt design for shear panels is summarized in Appendix C (Paragraph C18). Appendix C (Paragraph C18) also defines the minimum edge distance for anchor bolts based on ACE 355.1R-91.

[13] ACI 355.1R-91, Equation 3.2.